

BUCKLING OF STEEL AND COMPOSITE STEEL AND CONCRETE COLUMNS IN CASE OF FIRE

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Abstract. *Bare steel columns are known to have a very low fire resistance. The high thermal conductivity and the sudden decrease of the steel yield stress and Young Modulus in function of the temperature are responsible for this behaviour. The purpose of this study is to compare different modes of failure of building steel columns. The buckling of the columns is strongly influenced by the contact with other elements such as brick walls. Three types of situations were compared: steel columns embedded on brick walls, bare steel columns and composite columns made of partially encased steel sections. The deformed shapes of the columns are analyzed in this work.*

1 INTRODUCTION

The behavior of a steel and composite steel and concrete structure is strongly influenced by the high temperatures attained in the structural elements in case of fire. In columns the axial forces will increase up to a maximum and then reduce with their buckling. This increasing of axial forces result from the axial restraint imposed by the surrounding structure to the column subjected to fire and the decreasing from the degradation of the mechanical properties of steel and concrete with the temperature. This phenomenon has been studied for years, both experimentally and numerically, by several authors [4, 5].

Buckling can be substantially different when the column is completely engulfed in fire or embedded on a building wall. In the last case the high thermal gradients along the cross-section is responsible for high thermal stresses causing thermal bowing. An inversion of bending moments along the height of the column lead to lateral displacements moving the column from one to the other side.

In the Laboratory of Testing Structures and Materials of the University of Coimbra, Portugal, a large number of fire resistance tests were conducted in steel and composite steel and concrete columns, with restrained thermal elongation, embedded or not on brick walls [1, 2, and 3].

2 EXPERIMENTAL PROGRAMME

Figure 1 presents the test set-up, which were especially constructed in the Laboratory of Testing Materials and Structures of the University of Coimbra, for fire resistance tests on building columns embedded on brick walls (a) and isolated (b), both with restrained thermal elongation.

This system comprised a 3D restraining steel frame of variable stiffness (2) with the function of simulating the stiffness of the surrounding structure to the column subjected to fire. The use of a three-

dimensional frame allowed to take into account not only the axial stiffness but also the rotational stiffness, such as observed in a real structure. The restraining frame was composed by four columns, two upper beams and two lower beams, placed orthogonally. The beams of this frame were HEB 300 steel profiles, grade S355, in the test set-up for isolated columns (fig. 1(b)), and HEA200 in the test set-up for columns embedded on brick walls (fig. 1(a)). The connections between the structural elements of the restraining frame were performed with four M24 bolts, grade 8.8, except the connections between columns and the upper beams (2) where threaded rods M27, grade 8.8, were used. The stiffness of this restraining frame was varied from one to other test by changing the position of its columns.

In tests the columns were subjected to a constant compressive load that tried to simulate the serviceability load of the column when inserted in a real building. This load was 70% and 30% of the design value of buckling resistance at room temperature calculated according to Eurocodes 3 and 4 parts 1.1. [6, 7].

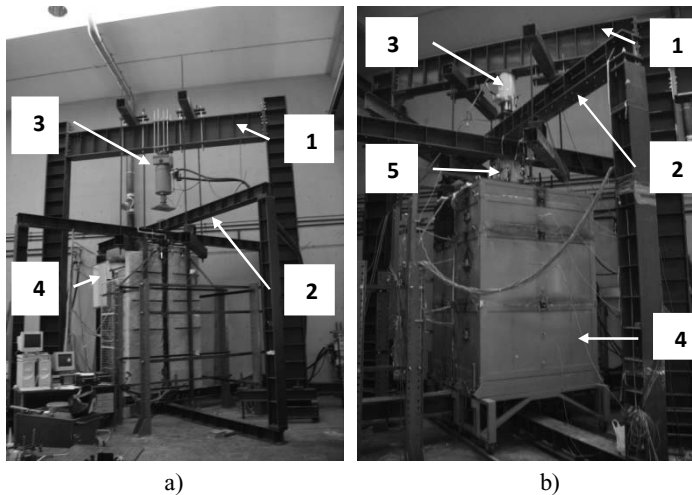


Figure 1: Experimental set-up a) columns embedded on walls b) isolated columns

This load was applied by a hydraulic jack of 3MN and was controlled by a load cell of 1MN placed between the upper beam of the 3D restraining frame and the hydraulic jack. The hydraulic jack was placed in a 2D restraining frame (1), in which was also placed a safety structure to prevent sudden collapse of the specimen.

In the test set-up for testing bare columns (fig. 1b), the thermal action was applied by a modular electric furnace (4). This furnace is composed by two modules of 1m and one module of 0.5m height, placed on top of each others, forming a chamber around the column of about 1.5m x 1.5m x 2.5m. In the testing set-up for columns embedded on walls, the thermal action was applied by a gas fired furnace, 2 meters height and 1 meter wide (4) (fig. 1a). Both furnaces followed approximately the standard ISO 834 fire curve.

A special device was built to measure the restraining forces generated in the testing column during the fire resistance test (5). It consists of a hollow and stiff steel cylinder, rigidly connected to an upper beam above the specimen. On top of the specimen was rigidly connected a massive steel cylinder that entered into the hollow cylinder. The massive cylinder was Teflon lined in order to prevent friction with the hollow cylinder. Inside the hollow cylinder there was a compression load cell of 3 MN to measure the restraining forces.

2.1 Steel columns embedded on brick walls

In this series, fourteen tests were performed, testing different orientations of the steel profile, with the web perpendicular and parallel to the wall surface, different thicknesses of the wall, two steel cross-sections, HEA200 and HEA160 (table 1). Eight tests were performed without loading. The tests with loading were performed with an applied load of 70% of the design value of the buckling load at ambient temperature, $N_{Rd,20}$. The thick wall was approximately of the same width of the steel profile width. The thin wall was approximately 75% wide of the steel profile width (table 1).

Table 1 – Test plan for steel columns embedded on walls

Test Number	Steel Profile	Orientation of web to furnace	Wall width (mm)	Load (kN)	Stiffness (kN/mm)	Type of buckling
E01	HEA 160	parallel	140	704 (70% $N_{Rd,20}$)	7	Bending
E02	HEA 160	perpendicular	140	704 (70% $N_{Rd,20}$)	7	Bending
E03	HEA 200	parallel	180	-	7	-
E04	HEA 200	perpendicular	180	-	7	-
E05	HEA 160	parallel	140	-	7	-
E06	HEA 160	perpendicular	140	-	7	-
E07	HEA 160	parallel	140	621 (70% $N_{Rd,20}$)	7	Bending
E08	HEA 200	parallel	140	-	7	-
E09	HEA 200	perpendicular	140	-	7	-
E10	HEA 160	parallel	100	-	7	-
E11	HEA 160	perpendicular	100	-	7	-
E12	HEA 160	parallel	100	621 (70% $N_{Rd,20}$)	7	Bending
E13	HEA 160	perpendicular	100	704 (70% $N_{Rd,20}$)	7	Bending
E14	HEA 160	parallel	100	1088 (70% $N_{Rd,20}$)	7	Bending

2.2 Steel bare columns

Table 2 describes the second series of fourteen tests, referring to isolated steel bare columns, totally engulfed in fire that resulted in a quite uniform heating within the cross section.

Table 2 – Test plan for isolated steel bare columns

Test Number	Steel Profile	Load (kN)	Stiffness (kN/mm)	Eccentricity	Type of buckling
E15	HEA 200	1000 (70% $N_{Rd,20}$)	13	Centered	local+global
E16	HEA 200	224 (70% $N_{Rd,20}$)	13	Two axis	local+global
E17	HEA 200	570 (70% $N_{Rd,20}$)	13	One axis	local+global
E18	HEA 160	621 (70% $N_{Rd,20}$)	13	Centered	global
E19	HEA 200	428 (30% $N_{Rd,20}$)	13	Centered	local+global
E20	HEA 160	266 (30% $N_{Rd,20}$)	13	Centered	global
E21	HEA 160	621 (70% $N_{Rd,20}$)	45	Centered	global
E22	HEA 160	266 (30% $N_{Rd,20}$)	45	Centered	global
E23	HEA 200	1000 (70% $N_{Rd,20}$)	45	Centered	local+global
E24	HEA 200	266 (30% $N_{Rd,20}$)	45	Centered	local+global
E25	HEA 200	428 (30% $N_{Rd,20}$)	128	Centered	local+global
E26	HEA 160	266 (30% $N_{Rd,20}$)	128	Centered	global
E27	HEA 200	1000 (70% $N_{Rd,20}$)	128	Centered	local+global
E28	HEA 160	621 (70% $N_{Rd,20}$)	128	Centered	global

Two different profile cross-sections, HEA200 and HEA160, leading two different slenderness, 50.6 and 63.3, two different load levels, 70% and 30% of $N_{Rd,20}$, and three different stiffness of the surrounding structure, 13, 45 and 128 kN/mm, were tested. The two other tests, E16 and E17 were performed only for HEA200, with the lower stiffness of the surrounding structure, 13kN/mm, but with eccentric loading: E16 with eccentricities of 0.2m in two directions and E17 with one eccentricity of 0.2m in one direction.

2.3 Composite steel and concrete columns

Table 3 describes the third series of twelve tests on partially encased H steel sections. The profiles cross-sections were HEA200 and HEA160, the load levels were 30 and 70% of $N_{Rd,20}$ [8, 9] and the stiffness of surrounding structure were 13, 45 and 128 kN/mm. Concrete C25/30 was used between flanges and the longitudinal reinforcement steel and the stirrups were A500. In this series no tests with eccentric loading were performed.

Table 3 – Test plan for composite steel and concrete columns

Test Number	Steel Profile	Load (kN)	Stiffness (kN/mm)	Reinforcement	Type of buckling
E29	HEA 160	261(30% $N_{Rd,20}$)	128	4 bars – 16mm + stirrups 6mm //0.15m	global
E30	HEA 160	610(70% $N_{Rd,20}$)	128	4 bars – 16mm + stirrups 6mm //0.15m	global
E31	HEA 200	508(30% $N_{Rd,20}$)	128	4 bars – 20mm + stirrups 8mm //0.15m	global
E32	HEA 200	1185(70% $N_{Rd,20}$)	128	4 bars – 20mm + stirrups 8mm //0.15m	global
E33	HEA 160	261(30% $N_{Rd,20}$)	45	4 bars – 16mm + stirrups 6mm //0.15m	global
E34	HEA 160	610(70% $N_{Rd,20}$)	45	4 bars – 16mm + stirrups 6mm //0.15m	global
E35	HEA 200	508(30% $N_{Rd,20}$)	45	4 bars – 20mm + stirrups 8mm //0.15m	global
E36	HEA 200	1185(70% $N_{Rd,20}$)	45	4 bars – 20mm + stirrups 8mm //0.15m	global
E37	HEA 160	261(30% $N_{Rd,20}$)	13	4 bars – 16mm + stirrups 6mm //0.15m	global
E38	HEA 160	610 (70% $N_{Rd,20}$)	13	4 bars – 16mm + stirrups 6mm //0.15m	global
E39	HEA 200	508(30% $N_{Rd,20}$)	13	4 bars – 20mm + stirrups 8mm //0.15m	global
E40	HEA 200	1185(70% $N_{Rd,20}$)	13	4 bars – 20mm + stirrups 8mm //0.15m	global

3 RESULTS

3.1 Steel columns embedded on walls

The steel columns embedded on walls had a differential heating characterized by greater temperatures on the exposed side, leading to huge thermal gradients within the cross-section. This difference of temperatures leads to a greater thermal expansion of the heated zone of the steel profile.

Due to the restraining prevented by the surrounding structure the thermal elongation was transformed in stresses, which being greater on the hot side, lead to bending moments, and the columns trends to bend instead of buckle.

The thermal bowing is a phenomenon in which the differential thermal action leads to an inversion of the deflection if the structural element, from one to the other side. This was observed in test E02, as showed in figures 2a) and 2b). This column has suffered at first, a deflection towards the side of the fire, and afterwards a deflection towards the opposite side, outside of the fire.

This is most likely to happen in steel columns with the web perpendicular to the wall surface, but it can also occur in case of the steel profile placed with the web parallel to the wall surface, as long as the upper beams are strong enough to withstand the applied load.

The differential heating provoked a redistribution of bending moments, moving from one to the other side, following the lateral displacements of the column.

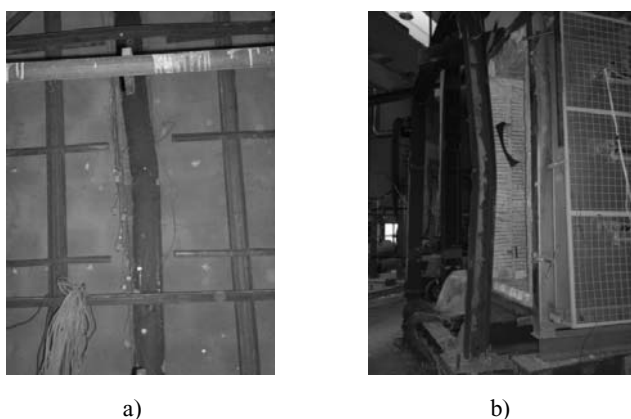


Figure 2: Column E02 after test - web perpendicular to wall – wall thickness 140mm - HEA 160

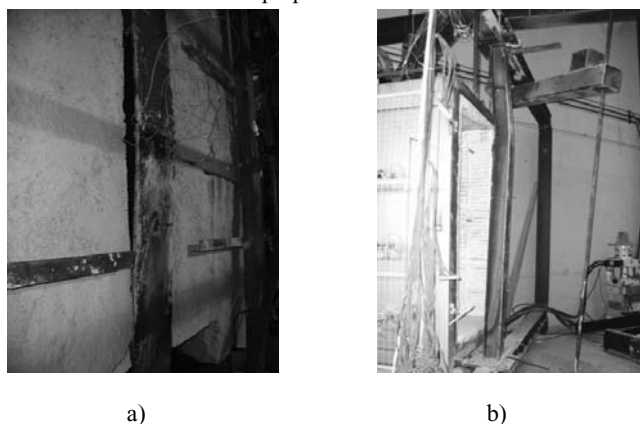


Figure 3: Column E13 after test - web perpendicular to wall – wall thickness 100mm - HEA 160

In figures 3a) and 3b), is observed column E13 after test. A HEA160, with the web perpendicular to the wall surface, is tested. The wall is 100mm thick, about 65% the column width. In this case, the

thermal gradient through the columns cross-section was not as strong as in test E02, leading to buckling in the plane of the wall. The wall plays an important role in preventing the sudden failure of the column.

Figure 4 presents the evolution of restraining forces versus mean temperature of the steel columns, for tests E07 and test E12. It is observed that the decreasing of restraining forces after the maximum is quite gentle meaning with this failure by bending. Also, it was concluded that the thicker walls provide a greater insulation to the steel profile, giving as result a lower increase of the restraining forces and higher fire resistance.

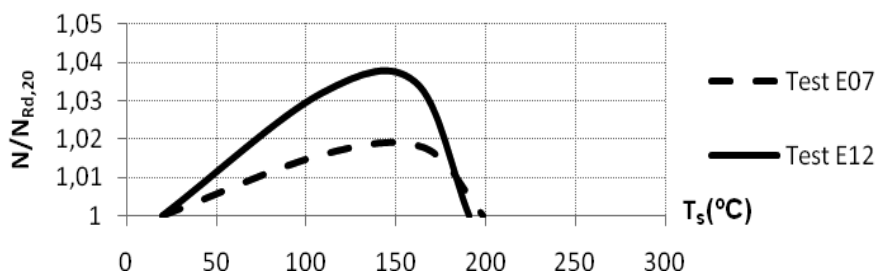


Figure 4: Restraining forces on columns embedded on walls in function columns mean steel temperature

3.2 Steel bare columns



a) local buckling



b) global buckling

Figure 5: Column E15 after test



Figure 6: Column E18 after test

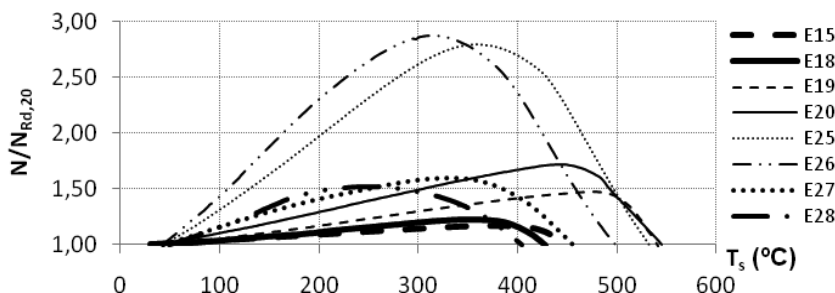


Figure 7: Restraining forces in bare steel columns in function of columns mean steel temperature

Despite of the detrimental effect caused by the contact with the walls, leading to bending moments to develop in the cross section of the columns, the uniform heating in bare steel columns leads to a faster heating of the cross section.

The buckling modes observed in the steel bare columns were global and local buckling for columns HEA200 (fig. 5a) and 5b)) and only global buckling for columns HEA160 (fig. 6). The columns presented in figures 5 and 6 were tested with the load level of 70% and stiffness of the surrounding structure of 13 kN/mm.

Figure 7 presents the evolution of restraining forces in bare steel columns in function of columns mean steel temperature. It can be observed that the lower load levels and higher the stiffness of surrounding structure, the higher the restraining forces. The restraining forces decrease in the major part of the cases not so sudden meaning global buckling of the columns.

3.3 Composite steel and concrete columns

In composite steel and concrete columns regardless the load level, type of cross section, and stiffness of surrounding structure, was only observed global buckling however with detachment of the concrete between flanges (fig. 8a)).

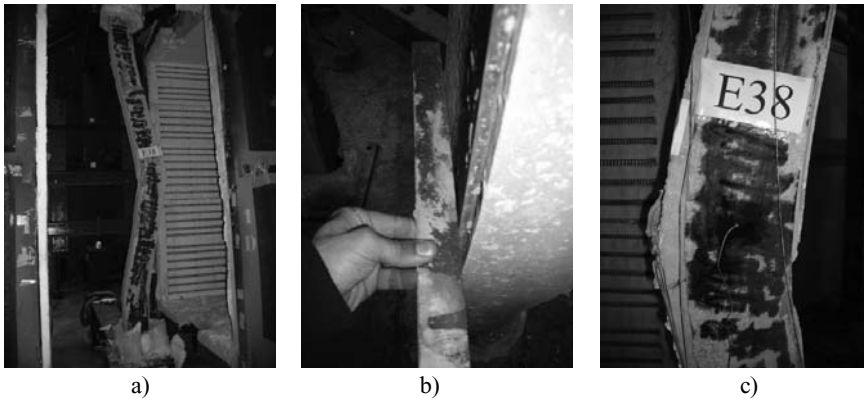


Figure 8: Column of test E38 after test

As it can be seen in figure 8b), the concrete between flanges prevented the buckling of these. The failure of some composite columns was observed with the detachment of the stirrups from the web allowing the concrete to move away as a block (fig. 8c)).

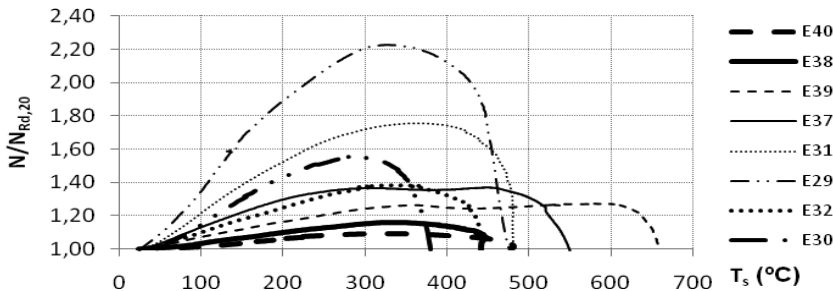


Figure 9: Restraining forces on composite columns in function of columns mean steel temperature

Figure 9 presents the evolution of restraining forces in the composite steel concrete columns in function of mean steel temperature. The conclusions are the same as for the steel columns. The lower the load level and the higher the stiffness of the surrounding structure the higher the restraining forces. A more pronounced descending of the restraining forces is observed in this case that could be associated to the detachment of the concrete between flanges.

4 CONCLUSIONS

The main conclusions regarding steel columns are related to thermal bowing. Columns in contact with walls under a fire event are submitted to a huge thermal gradient which is responsible for an inversion of displacements and bending moments from the heated to the unheated side of the column. The columns in this case bend instead of buckle. The walls have a favorable effect in preventing local buckling of column flanges and in case of columns with the web perpendicular to the wall, prevent buckling around minor axis, providing an increase on fire resistance.

Steel bare columns failed by buckling around minor axis. Local buckling was observed only in columns HEA200.

Composite steel-concrete columns were found to be very effective in resisting to fire. Besides having a higher load-bearing resistance, the concrete between flanges is effective in preventing local buckling and providing thermal insulation.

The load level has a great influence in the deflections in the instant of collapse. Higher load level leads to higher deflections for both types of columns.

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